A laboratory study on the undrained strength of a silty sand from Central Western Taiwan

Yao-Tao Huang, An-Bin Huang*, Yu-Chen Kuo, Ming-Dou Tsai

Department of Civil Engineering, National Chiao Tung University, 1001 Ta Hsueh Rd, Hsin Chu, Taiwan, ROC

Abstract

The sand deposit in Central Western Taiwan typically contained significant amounts of fines. The assessment of liquefaction potential using the simplified procedure often involved adjustment in field test results to account for the effects of fines. The available fines content (FC) adjustment methods are highly empirical and may lead to very different conclusions. The need and/or level of FC adjustment should be justified based on cyclic behavior of sand and its relationship with fines. This study made an attempt to provide that reference information for a silty sand commonly found in Central Western Taiwan. A series of isotropic consolidation, undrained monotonic and cyclic triaxial tests were conducted on reconstituted Mai Liao Sand (MLS) with various FCs and densities. Concurrent shear wave velocities were measured using bender elements in some of the cyclic triaxial tests. This paper describes the geological background, the laboratory tests and their implications in the assessment of liquefaction potential for MLS.
© 2004 Published by Elsevier Ltd.

Keywords: Sand; Cyclic resistance; Fines content; Critical state; Shear wave velocity

1. Introduction

Field test results such as the N value from standard penetration test (SPT), cone tip resistance (q_c) from cone penetration test (CPT) or shear wave velocity (V_s) have been used to assess the liquefaction potential for sand under the framework of simplified procedure [1]. The simplified procedure uses an empirical correlation between the cyclic resistance ratio (CRR) and the field test results as the principal criterion to determine if the soil is potentially liquefiable. These CRR correlation curves have been developed through field observations. Most of the available CRR correlations provide a clean sand base curve and a suggested procedure to account for fines (soil particles passing #200 sieve) content for sands that contain fines. Experience has indicated that mineral contents and plasticity of the fines are important contributing factors to the monotonic or cyclic behavior of a sand that contains fines [2]. The effects of plastic or non-plastic fines on the behavior of sand have been a subject of geotechnical research for many decades. These studies, however, have not lead to a consensus as to whether fines can increase or decrease the undrained strength or cyclic resistance of sand as described by Polito [3]. On the other hand, systematic

studies on the effects of fines on field test results such as N, q_c or V_s have rarely been reported. With these drawbacks, Youd et al. [1] emphasized that the CRR corrections based on fines contents (FCs) should be used with engineering judgment and caution.

The Chi Chi earthquake ($M_L = 7.3$, $M_W = 7.6$) of September 21, 1999 triggered extensive soil liquefaction in Central Western Taiwan. The majority of the sand deposit in this region was relatively compressible and had significant amounts of low to medium plastic fines. When performing back analysis of sand liquefaction potential in this region, the selection of field test method, its CRR correlation and the associated FC corrections could lead to significantly different results. The factors of safety (FS) against liquefaction from SPT were consistently higher than those from CPT, which in term were higher than those according to shear wave velocities [4,5]. In order to provide a basis for judging the correctness of the simplified liquefaction potential assessment procedure, it would be highly desirable to first establish an understanding of the undrained behavior of the silty sand in this region.

The authors performed a series of laboratory tests on reconstituted Mai Liao Sand (MLS), with various FC and densities. MLS was a typical silty fine sand from Central Western Taiwan. The laboratory tests included isotropic consolidation tests and consolidated undrained monotonic

* Corresponding author. Tel.: +886-35-722-803; fax: +886-35-734-116.
E-mail address: abhuang@mail.nctu.edu.tw (A.-B. Huang).

and cyclic shearing tests in a triaxial cell on specimens with FC = 0, 15, 30 and 50%. The triaxial cell was equipped with bender elements to allow shear wave velocity measurements on the soil specimens. The objective of these tests was to provide reference values on the compressibility, undrained shear strength and correlations between CRR and V_s for MLS. The effective confining stress and density conditions applied in the cyclic triaxial tests reflected the cases where we had the most interest in liquefaction potential analysis. This paper describes the geological background of MLS, the laboratory tests and their implications in the assessment of liquefaction potential for MLS.

2. Geological origin and physical properties of MLS

The source of MLS was the central mountain range that lied on the east side of Taiwan. Weathered sedimentary and metamorphic rocks such as sandstone, shale and slate on steep slopes were eroded by rainfall and transported by rapidly flowing streams before deposition on the west plain, to a thickness of several hundred meters. The process of transportation ground the fractured rock into sand and silt particles. Fig. 1 shows typical grain size distribution curves of MLS samples from the field. The specific gravity of clean MLS (particles retained on #200 sieve) had an average value of 2.69. Particles passing #200 sieve had an average specific gravity of 2.71. X-ray diffraction analysis of MLS showed significant amounts of muscovite and chlorite, in addition to quartz. Fig. 2 depicts scanning electron microscope photographs of the MLS particles retained on and passed #200 sieve. Particles retained on #200 sieve were angular and platy. The fines passing #200 sieve had a liquid limit of 32 and a plasticity index less than 8, which lied slightly below the A-line and classified as ML. The minimum void

Fig. 1. Grain size distribution of Mai Liao Sand.

ratios (e_{min}) were 0.646, 0.589, 0.593 and 0.564 and maximum void ratios (e_{max}) were 1.125, 1.058, 1.213 and 1.308 for MLS with 0, 15, 30 and 50% of FC, respectively. The e_{min} (maximum density) was determined according to ASTM D4253 method 1A using a standard 152.4 mm inside diameter compaction mold (total volume = 2830 cm³). The dry sand placed in the compaction mold was subjected to a surcharge of 14 kPa, and then electromagnetically vibrated under a 60 Hz frequency for 8 min. The e_{max} (minimum density) was obtained according to ASTM D4254 method C, using a 2000 glass graduated cylinder. Approximately 1000 g of dry sand was placed in the glass cylinder and then plugged with a stopper. The total volume under the loosest state was determined according to the height of the sand after swiftly tipping the cylinder upside down twice. It should be noted that none of the above ASTM standards were applicable for sand with fines in excess of 15%. Also, as will be described later, due to the compressibility of MLS, the relative density could only serve the purpose of initial specimen setup. The analysis of test data used void ratios to represent the soil density.

3. Specimen preparation

It is known that the specimen preparation method or soil fabric can have significant effects on the stress–strain relationship and cyclic strength of sand [2]. The situation is more complicated for silty sand as it involves additional issues of soil compressibility and particle segregation. According to Ishihara [2], the widest range in void ratio that can be attained is by means of moist tamping (MT), and the narrowest range by means of water sedimentation (WS). The specimen preparation method, however, is not expected to have significant effects on the critical state line (CSL) [2,6]. For tests on silty sand, Ishihara [2], Høeg et al. [7], and Vaid et al. [8] reported that MT was not able to create consistent specimens or to simulate the stress–strain response of natural alluvial soil deposits. Dry deposition (DD) and WS methods were used to form Tia Juana silty sand specimens [2] at the highest possible void ratio and then consolidated to the desired confining stress.

Soil specimens with FC of 0, 15, 30 and 50% by weight were used in the consolidation and monotonic triaxial tests in this study. In order to compare the undrained shear strength and other soil behavior indexes under a given state, it was necessary to create specimens with these FC and under the same void ratio. With all of its drawbacks, the authors chose MT as the main method for specimen preparation because of the wide range of FC involved and compressibility that varied with FC. As will be explained later, as the FC exceeded 30%, the range of possible void ratios became rather limited after the specimen was consolidated. In many cases, the specimens with FC over 30% had to be made under very loose conditions to compensate for its high compressibility so that
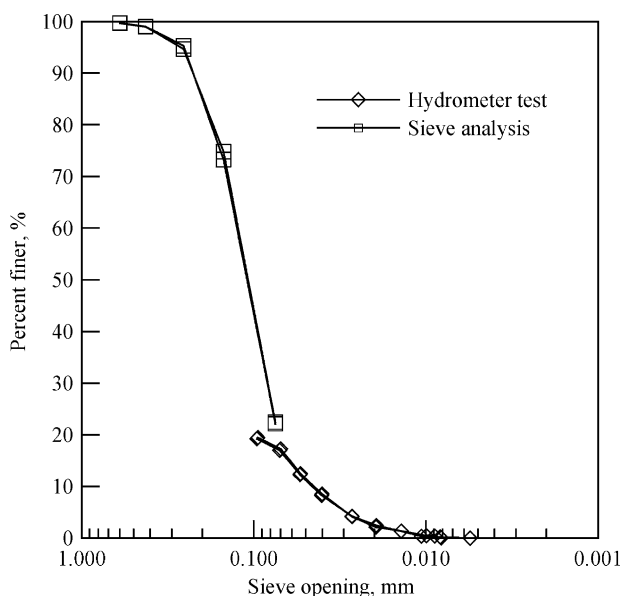
Retained on #200 sieve   Passed #200 sieve
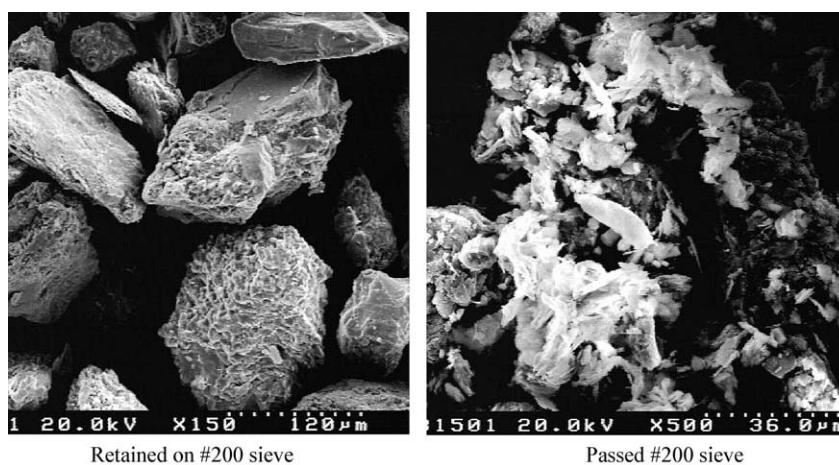
Fig. 2. Scanning electron microscope photograph of MLS particles.

the postconsolidation void ratio could be comparable to a specimen with FC less than 15%. MT was a practical and convenient way to create specimens with the necessary range of void ratios and thus was adopted to prepare most of the MLS specimens in this study. Recognizing the potential problems of MT specimen preparation method, the soil behavior correlations were analyzed internally among laboratory test data only. In addition, a series of tests were conducted to verify the uniformity and consistency of the specimens prepared by MT and to compare with those by DD and WS methods as will be presented later.

A large quantity of MLS taken from the field was washed through a #200 sieve and then oven dried to separate the fines. The coarse (retained on #200 sieve) and fine (passing #200 sieve) particles were then blended together to obtain the desired FC. For consolidation and monotonic triaxial tests, the specimens had a diameter of 75 mm and length of 150 mm. The cyclic triaxial test specimens had a diameter of 50 mm and length of 100 mm. The test specimens were created following the MT method in four layers. The sand was mixed with a moisture content (w) of 8% and then placed in the split mold and compacted to the desired density. Because of capillary, there was no need to apply vacuum when removing the split mold. The specimen was permeated with CO_2, flushed with de-aired water and saturated under a back-pressure of 300 kPa. All specimens had a minimum pore pressure parameter B of 0.95.

It should be emphasized that due to the nature of mineral contents, MLS was relatively compressible. The change of void ratio between specimen setup and after the application of confining stress in the triaxial cell could be very significant. During the consolidation tests, the amount of pore water being absorbed or squeezed out of the saturated specimen was monitored. At the end of consolidation, the entire specimen was used to measure its moisture content and the postconsolidation void ratio (e) was calculated from this moisture content. The e values at earlier stages of consolidation were then back calculated according to the pore water volume changes recorded during the test.

The end-of-consolidation void ratio was used in reporting the triaxial test results. A series of isotropic consolidation tests were performed first to establish the stress–volume relationship under various initial relative densities (D_{r0}). For FC = 0 and 15%, D_{r0} ranged from 0 to 85%. As FC reached 30%, the consolidation test results became rather inconsistent and not repeatable when D_{r0} was less than 50% or larger than 85%. For this reason, the isotropic consolidation tests on specimens with FC = 30 and 50% were limited to the D_{r0} of 50 and 70%.

In preparing the monotonic and cyclic triaxial specimens, a D_{r0} was chosen so that the desired initial state could be reached after application of the confining stress as predicted by the isotropic consolidation tests. Following this procedure, the specimen was always denser upon application of the confining stress than it was after the specimen preparation. The final void ratio was independently determined by measuring the water content after the triaxial test similar to the case of consolidation tests.

4. Effects of specimen preparation procedure

MLS with the desired target FC was mixed first in dry condition. For specimens prepared by MT, the specimens were made following the procedure described above. The specimen prepared by DD was carried out using a known weight of dry sand that was poured from a minimum height into the rubber membrane lined split mold with the aid of a funnel. The WS method was similar to DD, except that the sand was deposited through de-aired water. The specimen was saturated and back-pressured as described earlier. Upon saturation and consolidation to an effective confining stress of 100 kPa, the specimen was removed from the triaxial cell and sliced horizontally into five equal layers, numbered 1–5 from top to bottom. For each slice, the water content was measured and FC determined by wash sieving. Table 1 shows the test results and statistics from MT specimens with target FC of 0, 15, 30 and 50%, and specimens prepared by

Table 1
Water content and fines content distributions within soil specimens

Target FC	D_{r0}	e		Level 1	Level 2	Level 3	Level 4	Level 5	Mean	Standard deviation
Moist tamping										
0	70	0.82	FC	3.90	4.25	2.39	3.99	1.63	3.23	1.15
			w	30.56	30.49	30.09	30.84	30.97	30.59	0.34
15	50	0.82	FC	17.54	16.04	15.95	16.67	17.10	16.66	0.68
			w	30.59	30.51	30.46	30.63	30.65	30.57	0.08
30	65	0.77	FC	30.07	31.45	31.67	30.11	29.44	30.55	0.96
			w	28.56	28.49	29.87	27.38	28.93	28.65	0.90
50	50	0.71	FC	51.46	51.15	50.79	50.85	51.31	51.11	0.29
			w	26.82	26.56	26.12	26.13	26.79	26.48	0.34
Water sedimentation										
30	65	0.76	FC	31.63	26.38	27.94	28.47	28.53	28.59	1.91
			w	26.24	28.43	27.39	30.00	28.82	28.18	1.43
Dry deposition										
30	65	0.69	FC	28.68	27.47	29.28	27.97	28.00	28.28	0.71
			w	25.46	26.30	25.76	25.73	24.71	25.59	0.58

DD and WS method with a target FC of 30%. The results showed that variations of FC or w within the specimens were mostly random indicating that there were no signs of migration of fines in the specimens, regardless of the specimen preparation method. For specimens of FC = 30%, the specimen prepared by WS had the lowest uniformity whereas the DD specimen was the most uniform. The standard deviation of water contents of the WS specimen represent a coefficient of variation of approximately 5%, well within the accuracy of water content measurements. The results described above and shear wave velocities measured within the triaxial specimens to be presented later show that the specimens prepared by MT are reasonably uniform and consistent.

5. The stress–volume ratio and strength behavior of MLS under monotonic loading conditions

A Wykeham–Farrance triaxial apparatus was used in the monotonic consolidation and triaxial shearing tests. An internal load cell was used to monitor the axial force imposed on the specimen and an externally mounted LVDT was used to measure the axial deformation. Fig. 3 shows the relationship between e and effective mean normal stress $p'(= (\sigma'_v + 2\sigma'_h)/3)$, where σ'_v is the effective vertical stress and σ'_h, the effective horizontal stress, from the isotropic consolidation tests. The e_{min} and e_{max} are marked in the figure. In general, the compressibility increased with FC and p', and decreased as the specimen became denser. When the FC reached 50%, the application of initial confining stress and the saturation process apparently caused enough compression that the consolidation curves were almost indistinguishable among specimens with different D_{r0}. In many cases, the e became less than e_{min}, or the postconsolidation relative density exceeded 100%.

Isotropically consolidated undrained triaxial tests were performed on specimens with FC = 0, 15, 30 and 50%. The void ratios applied corresponded to loose to dense conditions. The specimens were sheared monotonically by axial compression to strains well in excess of 20% to reach or be close to critical state [6]. For the densities and confining stresses applied, the specimens rarely demonstrated dilatant behavior, even for dense specimens. In many cases, the test results showed strain softening behavior. The strain softening seemed to be more apparent as the FC increased. The relationships among deviator stress, excess pore pressure and axial strain, as well as the effective stress paths in terms of $q(= (\sigma'_v - \sigma'_h)$ and $p'(= (\sigma'_v + 2\sigma'_h)/3)$ from all the consolidated undrained triaxial tests are depicted in Fig. 4a–c. A combination of deviator stress, q and pore pressure was chosen for each undrained triaxial test according to the definition of critical state where the sand deformed continuously at relatively constant pore pressure and constant q. The critical state points are marked in Fig. 4a–c. A straight line was fitted to the data set of (p', q) that corresponded to the critical state of each triaxial test as shown in Fig. 4c. The slope of this fitted line was referred to as M_s. For the tests performed, $M_s = 1.24$ for FC = 0–30%, and $M_s = 1.21$ as FC reached 50%. These $M_s(= 6 \sin \phi'_s/(3 - \sin \phi'_s))$ values should correspond to interparticle friction angles (ϕ'_s) of 30.9 and 30.2°, respectively. The similarity in ϕ'_s regardless of the FC indicates that the coarse and fine MLS particles had similar grain to grain frictional behavior.

A CSL was established by fitting a line to the data set of p' at the critical state against the void ratio for each of the FC tested as shown in Fig. 5. The CSL became curved as p' approached 100 kPa. For FC $\leqq$ 30%, the coefficient of correlation of the CSL was over 0.85. As FC reached 50%, the coefficient of correlation of the CSL was reduced to 0.37. Because of this lack of consistency in monotonic
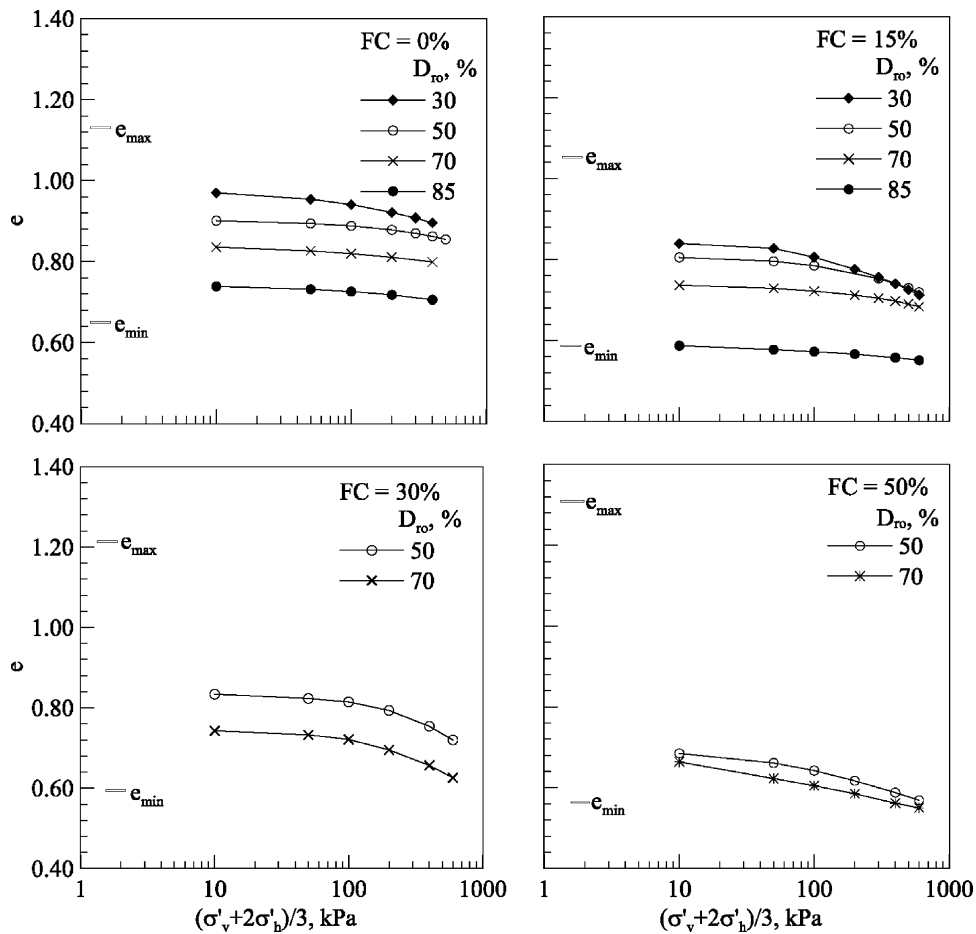
Fig. 3. Compressibility of MLS.

triaxial tests, no cyclic triaxial tests on specimens with FC = 50% were conducted.

6. The cyclic resistance of MLS

The CRR values were measured using a stress controlled pneumatic cyclic triaxial apparatus made by Seiken Inc. of Japan. The soil specimen was consolidated under an isotropic effective confining stress σ'_c and then subjected to a cyclic deviator stress, σ_d in axial direction. Three to five cyclic triaxial tests were performed with various $\sigma_d/2\sigma'_c$ values. CRR was defined as the $\sigma_d/2\sigma'_c$ that produced an axial strain of 5% in double amplitude in 20 cycles (N_c) of uniform load application. Since in the simplified procedure, CRR is usually correlated with in situ test results normalized with respect to an effective overburden stress of one atmospheric pressure or 100 kPa, all CRR measurements on MLS were thus, conducted with $\sigma'_c = 100$ kPa isotropically applied to the soil specimen during consolidation. The FC were limited to 0, 15 and 30%. Fig. 6 depicts the CRR versus N_c from cyclic triaxial tests.

An attempt was made to compare CRR for MLS with different FC but similar void ratio. Because of the drastically different range of void ratios under various FC as shown in Fig. 3, such a comparison would be rather difficult. Void ratios of approximately 0.80 (close to the maximum applicable e for specimens with FC = 30%) and 0.73 (close to the minimum applicable e for specimens with FC = 0%) represent the maximum and minimum void ratio, respectively, that could be used as a common value for all three FC applied in the cyclic triaxial tests. According to Fig. 6, CRR consistently decreased as FC changed from 0 to 15 and 30%, for tests conducted under common void ratios of 0.80 and 0.73. It is conceivable that similar trend would exist for void ratios between 0.80 and 0.73.

The data points of CRR versus state parameter (difference in void ratio between the initial state and critical state under the same effective mean normal stress) appear to have a consistent trend, regardless of the FC as shown in Fig. 7. The straight line in Fig. 7 had a coefficient of correlation of 0.81 with the data points. This correlation may be used to provide a first order estimate of CRR based on state parameter for MLS with various FC within the range of 0 and 30%. There is a theoretical basis for this correlation to be valid as the state parameter relates to soil dilatancy.

7. Shear wave velocity measurements in triaxial specimens

Bender elements of the type described by Dyvik and Madshus [9] were installed in the top and bottom platens of the cyclic triaxial cell, each projecting 3 mm into the soil sample. The shear wave velocity, V_s, was measured using the bender elements after the specimen was consolidated, prior to the cyclic triaxial test. A single sinusoidal pulse with a frequency of 7–10 kHz and amplitude of ± 10 V was
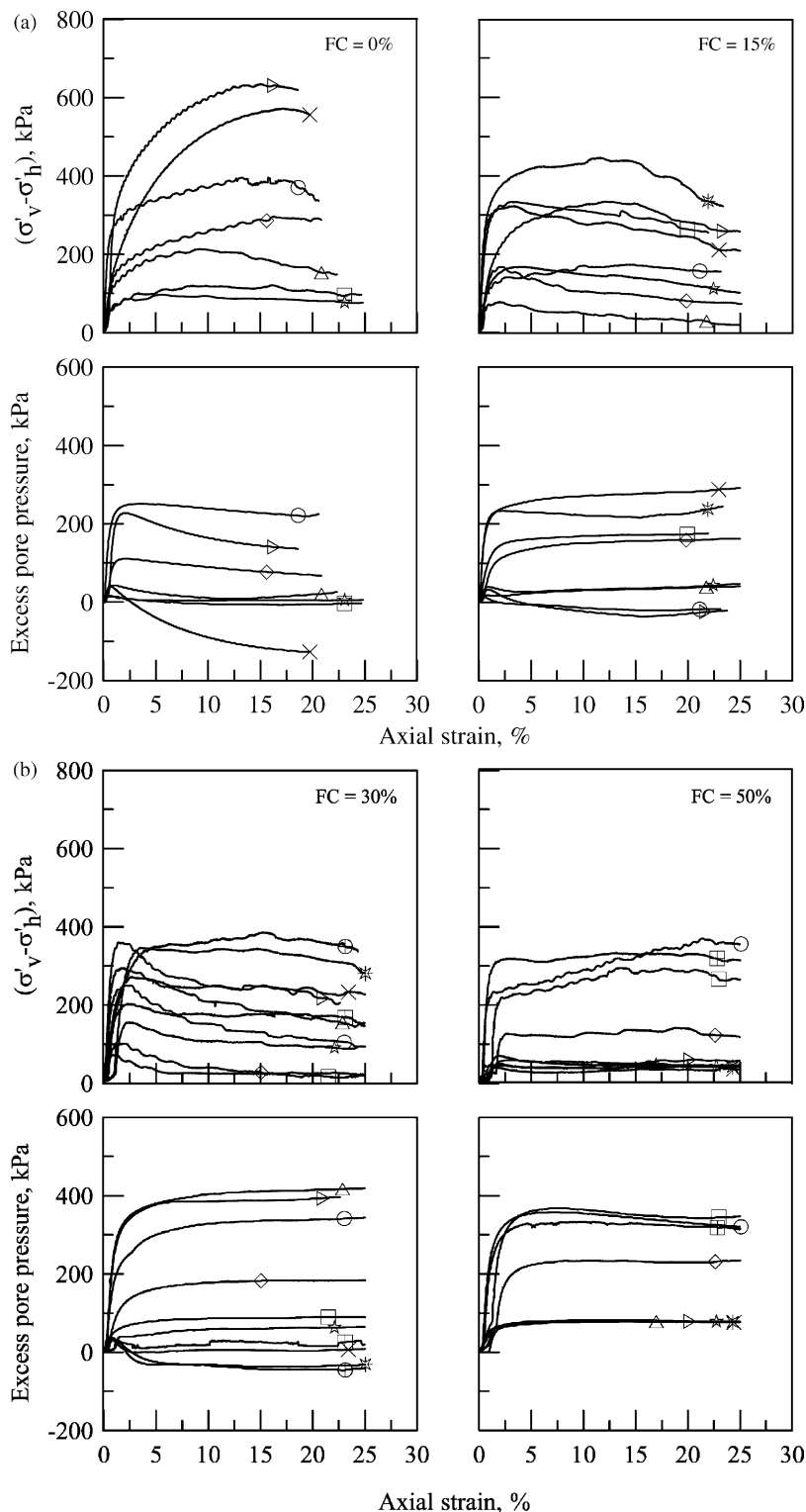

Fig. 4. (a) Stress, excess pore pressure and strain relationships from tests with FC = 0 and 15%. (b) Stress, excess pore pressure and strain relationships from tests with FC = 30 and 50%. (c) Effective stress paths from triaxial tests.
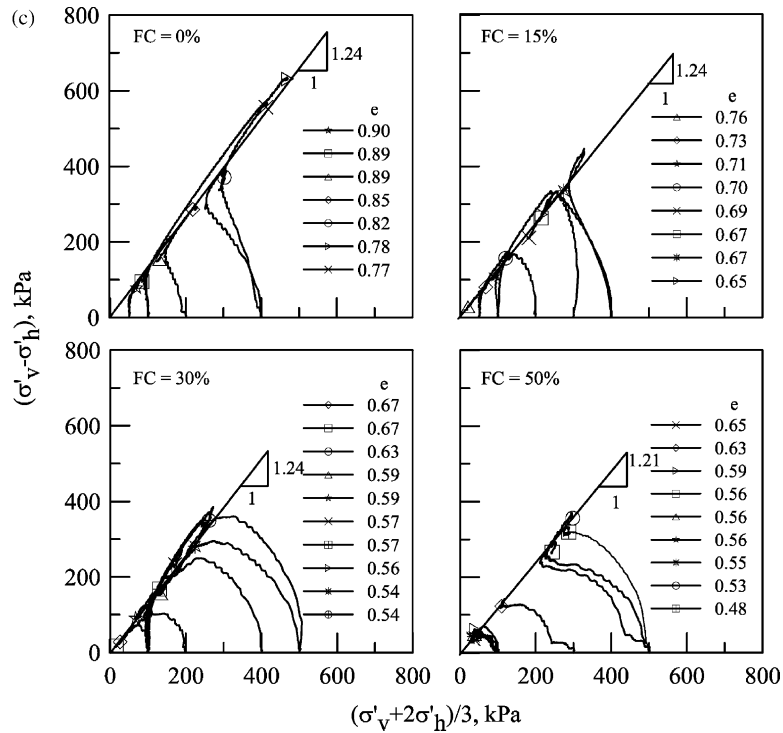
Fig. 4 (*continued*)

applied in the bender element tests. Fig. 8 shows an example of a time history of output voltage from the receiver. In this bender element test, the specimen with FC = 15% and $D_{r0} = 79\%$ was consolidated under an effective stress of 100 kPa, and the sinusoidal pulse frequency was 7 kHz. The determination of the shear wave travel time followed a procedure suggested by Kawaguchi et al. [10]. Point C marked in Fig. 8 was selected as the shear wave arrival time.
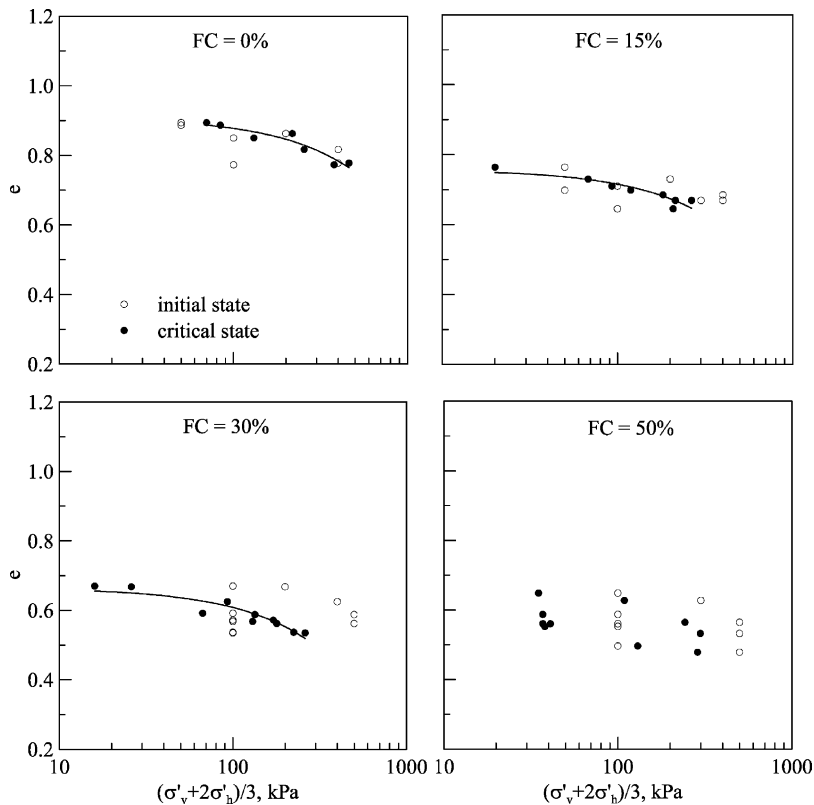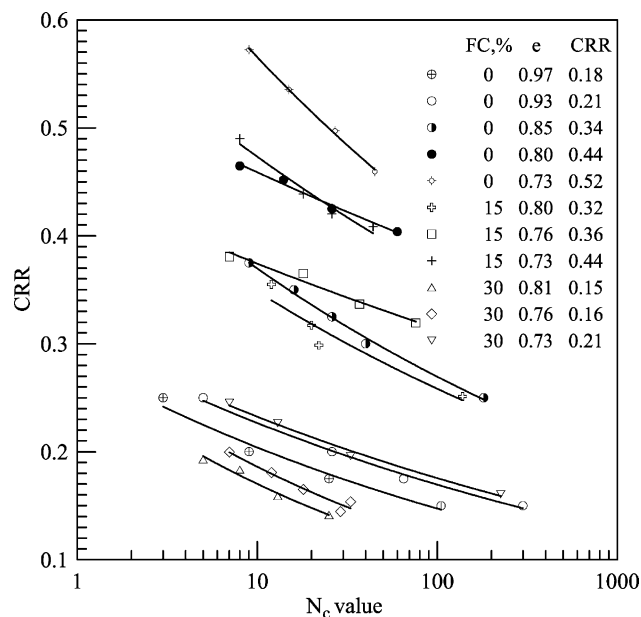

Fig. 5. Critical state lines of MLS.

FC,%	e	CRR
⊕	0	0.97	0.18
○	0	0.93	0.21
◑	0	0.85	0.34
●	0	0.80	0.44
◇	0	0.73	0.52
⊕	15	0.80	0.32
□	15	0.76	0.36
+	15	0.73	0.44
△	30	0.81	0.15
◇	30	0.76	0.16
▽	30	0.73	0.21

Fig. 6. CRR versus N_c.

Fig. 8. Time history of output voltage from the receiver.

The tip-to-tip distance between the two bender elements was used as the travel distance in the calculation of shear wave velocity.

Fig. 9 plots the relationships between e and V_s according to bender element measurements, under various FC. According to Fig. 9, for similar e, V_s decreases as FC increases. The results in Fig. 9 shows that for specimens with the same FC and similar e, the measured V_s clustered within a rather narrow range. This is a further indication that the specimen preparation by MT and V_s measurement procedures were sufficiently repeatable.
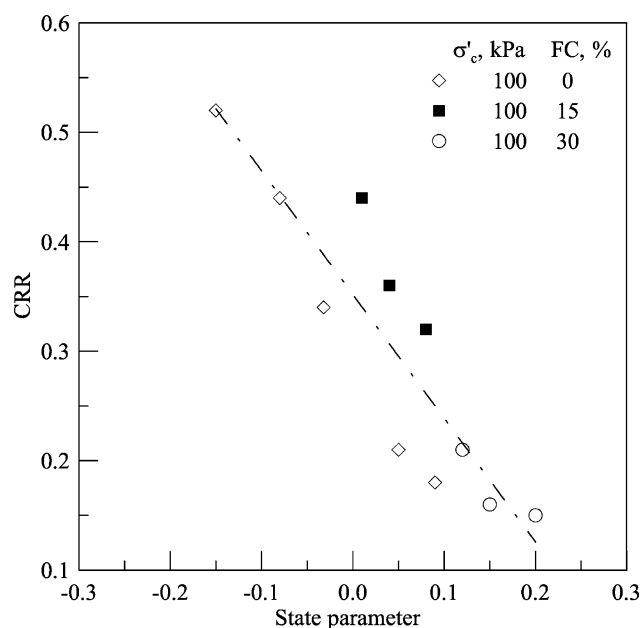
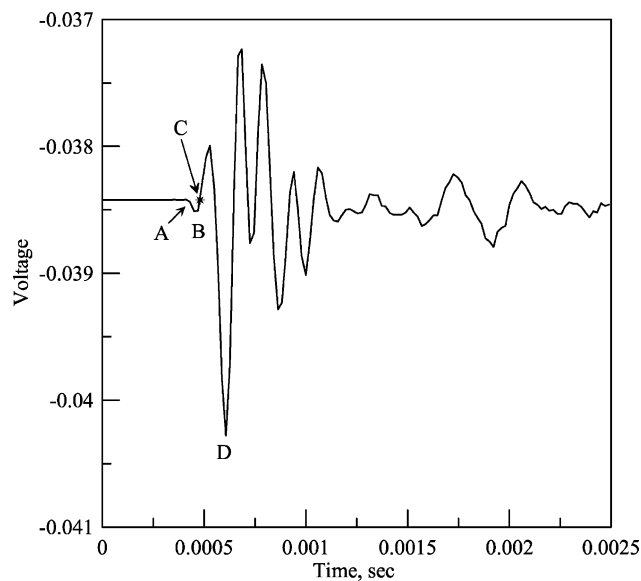Following the expression proposed by Hardin and Richart [11], V_s could be related to e as

$$V_s = (a - 44.6e)(\sigma_c')^{0.25} \qquad (1)$$

where $a = 91$, 85 and 79 for FC $= 0$, 15 and 30, respectively.

V_s is in m/s and σ_c' in kPa. The coefficient of correlation between Eq. (1) and the corresponding data points were 0.93, 0.79 and 0.97, respectively, for FC $= 0$, 15 and 30%. According to data presented in Fig. 9 and Eq. (1), the V_s of MLS was at least 30% smaller than those of Ottawa sand under similar states as reported by Hardin and Richart [11].
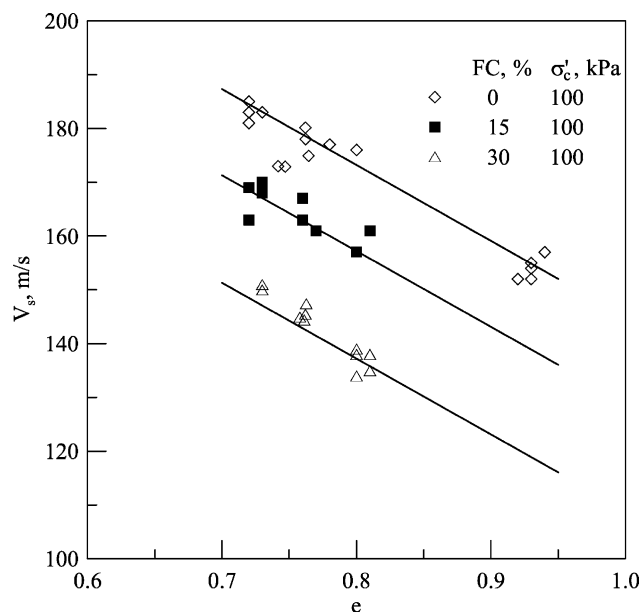

σ'c, kPa	FC, %
◇	100	0
■	100	15
○	100	30

Fig. 7. CRR versus state parameters.

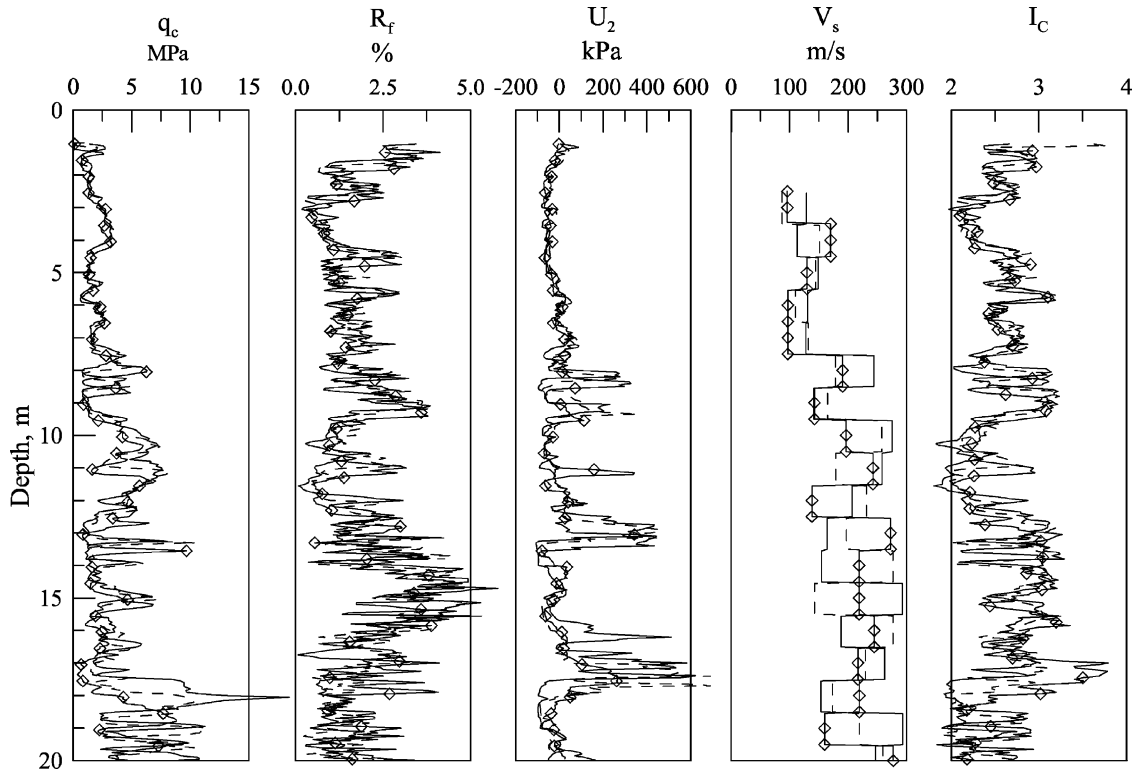| FC, %	σ'c, kPa
◇	0	100
■	15	100
△	30	100

Fig. 9. V_s versus e.

Fig. 10. SCPT test results from Yuan Lin test site.

There was a significant decrease of V_s as the FC increased from 0 to 30%, indicating that the fine grains of MLS were softer under small strains. These V_s values are comparable to those from seismic cone penetration tests (SCPTs) performed in silty sand with similar geological background, at a test site in Yuan Lin which was approximately 40 km east of the origin of MLS. Profiles of cone tip resistance (q_c), sleeve friction ratio (R_f), excess pore pressure readings immediately behind the cone tip (U_2), V_s and soil behavior index (I_C) from three SCPT performed within a distance of 5 m are shown in Fig. 10. The V_s values within the top 10 m, varied mostly between 100 and 200 m/s.

8. Relationships among CRR, fines content and V_s

Since for every cyclic triaxial test specimen, there was a V_s taken prior to the cyclic loading test, a correlation between laboratory determined CRR and V_s could be easily established based on these data. The shear wave velocity corrected to an overburden stress of 1 atmospheric pressure (P_a) or V_{s1} is the same as V_s from bender element measurements since all V_s measurements and cyclic triaxial tests were performed with $\sigma'_h = \sigma'_v = 100$ kPa $= 1P_a$. Fig. 11 shows a plot of CRR versus V_{s1} from this study and their comparison with the correlations proposed by Andrus and Stokoe [12]. For a given CRR, the V_{s1} value from laboratory tests on MLS specimens were generally smaller than those suggested by Andrus

and Stokoe [12]. There were no significant differences in the CRR–V_{s1} correlation as FC increased from 0 to 30%. The result may offer an explanation for the reported low FS against liquefaction using shear wave measurements in this region [4,5].

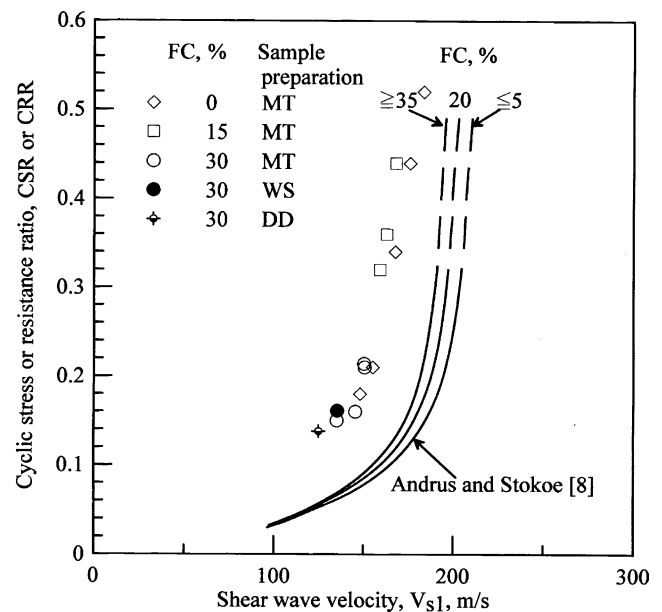It should be emphasized that the findings may be soil specific and strongly influenced by the specimen
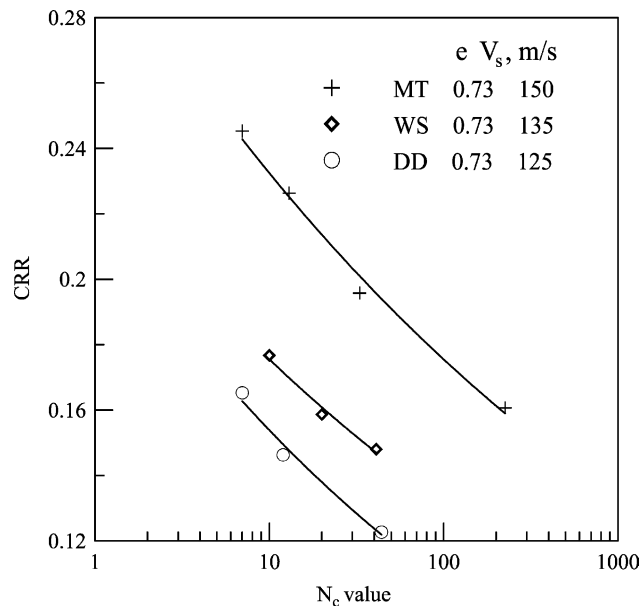

Fig. 11. CRR versus V_{s1}.

Fig. 12. Effects of specimen preparation method on CRR.

preparation and test procedures. The authors are not ready to suggest that the CRR–V_{s1} correlation as proposed by Andrus and Stokoe [12] should be revised for field applications in this region, solely based on the data reported herein. The evidence was strong enough, however, to indicate that the Andrus and Stokoe [12] correlation may be too conservative and further evaluation of the field data is warranted for the silty sand in this region. The test data also raised a question as to the necessity of FC adjustment for the CRR–V_{s1} correlation.

A limited number of cyclic triaxial and bender element tests were performed on specimens with FC = 30% and prepared by DD and WS methods. Fig. 12 depicts the CRR versus N_c from this series of cyclic triaxial tests. According to Fig. 12, for e of 0.73, specimens prepared by DD had the lowest cyclic strength and those by MT had the highest. Similar comparisons were also reported by Ishihara [2]. V_s measurements followed the same trend as the CRR. As a result, the additional CRR–V_{s1} data points from the WS and DD specimens fit well within the trend obtained from specimens prepared by MT method as shown in Fig. 11.

9. Concluding remarks

Due to the mineral and grain characteristics, the compressibility of MLS was relatively high and the CSLs were curved even under mild confining stresses. The laboratory measured V_s values obtained in MLS were smaller in comparison with those in reconstituted clean, quartz dry sand under similar states. These lower values are most likely related to the relatively soft and crushable

nature of MLS and the effects of fines. The grain characteristics of MLS could also lead to lower CRR from cyclic triaxial tests. Apparently, the decrease in CRR was not as significant as that in V_s according to the data presented in this paper. As a result, the CRR–V_s correlations developed from laboratory tests on MLS fell to the left of the correlation proposed by Andrus and Stokoe [12]. For void ratios ranging from 0.73 to 0.80 and FC at 0, 15 and 30%, the test data in this paper seem to indicate that the CRR decreases with FC for MLS under the same state. A similar trend was found between the V_s and FC.

The laboratory test results presented in this paper may help in explaining why FS against liquefaction from shear wave velocity were consistently low for the silty sand in Central Western Taiwan. It is well recognized that the soil fabric and aging can affect CRR and V_s. MLS specimens with 30% of FC, prepared by three different methods resulted in significantly different CRR values. The soil specimens prepared by MT method may not duplicate the soil compressibility and fabric in the field. It is thus, premature to conclude that the currently used CRR–V_s correlation should be modified solely in view of the new laboratory data. The effects of grain characteristics are not strongly influenced by soil fabric and aging. The data presented in the paper did indicate that the effects of grain characteristics can be very significant and should be carefully considered in the assessment of liquefaction potential for the silty sand in Central Western Taiwan.

References

[1] Youd TL, Idriss IM, Andrus RD, Arango I, Castro G, Christian JT, Dobry R, Liam Finn WD, Harder Jr LF, Hynes ME, Ishihara K, Koester JP, Liao SSC, Marcuson III WF, Martin GR, Mitchell JK, Moriwaki Y, Power MS, Robertson PK, Seed RB, Stokoe II KH. Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. J Geotech Geoenviron Eng, ASCE 2001;127(10):817–33.

[2] Ishihara K. Liquefaction and flow failure during earthquakes. Geotechnique 1993;43(3):351–415.

[3] Polito CP. The effects of nonplastic and plastic fines on the liquefaction of sandy soils. PhD Thesis, Virginia Polytechnic Institute and State University, Blacksburg, Virginia, 1999.

[4] Moh and Associates, Soil liquefaction and rebuild policy evaluations, Phase I (Yuan Lin, Ta Tzuen and Tou She Townships (in Chinese)), Taipei, Taiwan: Moh and Associates; 2000.

[5] Huang AB, Lee DH, Lin PS, Tsai JS, Lin CP, Chi YY, Ku CS, Chen JW, Juang CH, Liou YJ. In: Culligan, et al., editors. Lessons learned from the post Chi Chi EQ geotechnical explorations in Central Western Taiwan. In: 12th Pan American Conference on Soil Mechanics and Geotechnical Engineering, Cambridge, Massachusetts, vol. 1.; 2003. p. 245–52.

[6] Been K, Jefferies MG, Hachey J. A critical state of sand. Geotechnique 1991;41(3):365–81.

[7] Høeg K, Dyvik R, Sandbaekken G. Strength of undisturbed versus reconstituted silt and silty sand specimens. J Geotech Geoenviron Eng 2000;126(7):606–17.

[8] Vaid YP, Sivathayalan S, Stedman D. Influence of specimen-reconstituting method on the undrained response of sand. Geotech Test J 1999;22(3):187–96.

[9] Dyvik R, Madshus C. Laboratory measurement of G_{max} using bender element. Proceedings of ASCE Annual Convention: Advance in the Art of Testing Soils under Cyclic Conditions, Detroit. 1985. p. 186–96.

[10] Kawaguchi T, Mitachi T, Shibuya S. Evaluation of shear wave travel time in laboratory bender element test. Proceedings of the 15th International Conference on Soil Mechanics and Geotechnical Engineering, Istanbul 2001;1:155–8.

[11] Hardin BO, Richart FE. Elastic wave velocities in granular soils. J Soil Mech Found Eng Div, ASCE 1963; 89(SM1):33–65.

[12] Andrus RD, Stokoe II KH. Liquefaction resistance of soils from shear-wave velocity. J Geotech Geoenviron Eng, ASCE 2000;126(11): 1015–25.